

APPENDIX C

Preliminary Drainage Study

**CEDAR GROVE APARTMENTS
AFFORDABLE HOUSING PROJECT
EIS/EIR**

**PRELIMINARY DRAINAGE STUDY
PROPOSED PROJECT**

Placer County, California

August 2004



JWA CONSULTING ENGINEERS, INC.

Pleasant Hill, CA ■ Zephyr Cove, NV

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EXECUTIVE SUMMARY

A drainage analysis for the 10-year and 100-year storm frequencies was developed for the Cedar Grove Apartments Affordable Housing Project for the existing and Proposed Project conditions. Since the total watershed area at this site is less than 200 acres, the 10-year and 100-year storm flows were calculated based on the peak calculation method described in the October 1997 Addendum to the Placer County Flood Control and Water Conservation District Stormwater Management Manual (SWMM). Figures 1.1 and 1.2 are vicinity and location maps respectively.

The peak flow generated by the 10-year storm frequency was calculated to be 13.3 cubic feet per second (cfs) for the existing condition. For the proposed conditions, the peak flow generated by the 10-year storm frequency was calculated to be 13.5 cfs. The total flow generated by the 100-year storm frequency was calculated to be 21.7 cfs for the existing condition. For the proposed conditions, the total flow generated by the 100-year storm frequency was calculated to be 22.0 cfs. Therefore, the proposed improvements result in a minor increase in the rate and volume of flow generated during the 10- and 100-year events, 1.5% and 1.4% respectively. However, the improvements required to treat and retain the runoff volumes contributed by the 20-year, one-hour storm will decrease the flows associated with the proposed site improvements to levels which will likely be less than the existing conditions.

1. INTRODUCTION

1.1 General

The purpose of this preliminary drainage study is to evaluate the drainage aspects of the improvements in the proposed alternative for the Cedar Grove Apartments Affordable Housing Project with the results incorporated within the analysis performed by the EIS/EIR preparation by the EDAW team for the Placer County Planning Department, the California Environmental Quality Act (CEQA) and the Tahoe Regional Planning Agency (TRPA). This report has been prepared in accordance with the requirements of the Stormwater Management Manual (SWMM) developed by the Placer County Flood Control and Water Conservation District. The project must also demonstrate compliance with the California Water Quality Control Board, Lahontan Region (Lahontan), the National Pollution Discharge Elimination System (NPDES) General Permit, and the TRPA Code of Ordinances. Therefore, the report also discusses treatment, through use of water quality best management practices (BMPs), of the runoff associated with the 20-year, one-hour storm event as directed by the permitting agencies.

1.2 Project Location

The project area is located in Tahoe Vista, California north of State Route 28, and west of National Avenue in Tahoe Vista. The project area is directly south of the North Tahoe Regional Park and north of the adjacent Mourelatos Resort. A vicinity map showing the project location is included as Figure 1.1. The project area is shown on a portion of the Kings Beach 7.5 minute U.S.G.S. quadrangle map in Figure 1.2.

1.3 Project Watershed Description

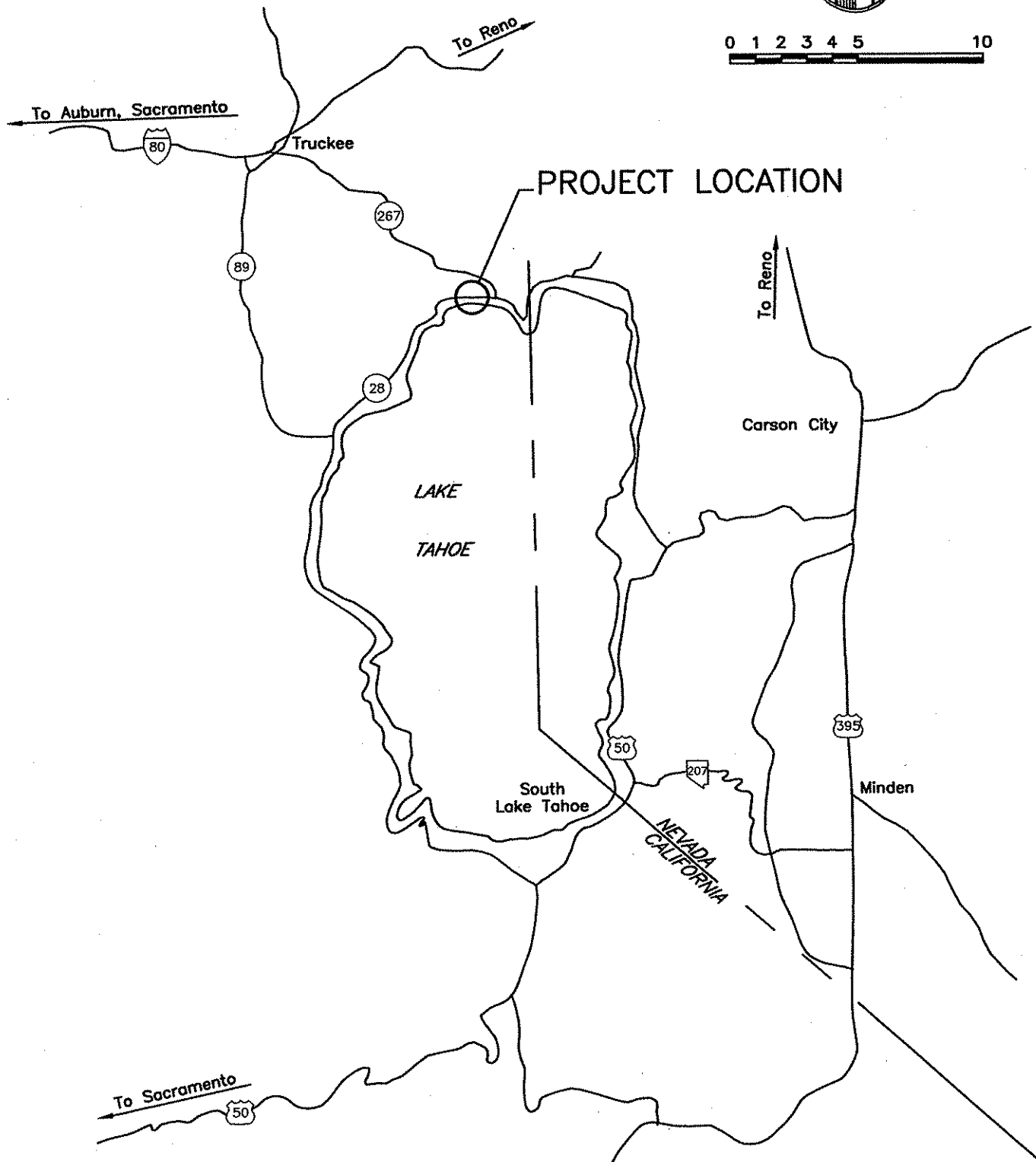
An area to the west and northwest contributes runoff to the project area by means of overland flow and disconnected overland flow across the Tahoe Estates Subdivision. The Tahoe Estates Subdivision does not have a designated discharge point to the parcel but contributes runoff from both pervious and impervious surfaces such as driveways and roadways to the project area via overland flow. A small portion of the North Tahoe Regional Park area in the northernmost portion of the watershed also contributes offsite runoff to the watershed. The Watershed Map, Figure 1.3, shows the tributary area, soils types, and topography, and provides direction arrows for the 100-year overland flow path.

Based on the Soil Survey for the Tahoe Basin Area, California and Nevada (Soil Survey), by the USDA Soil Conservation Service and Forest Service of March 1974, the project watershed contains soil Type JwD (very stony sandy loams) and JwE (very stony sandy loams) and consist of stony soils underlain by basic volcanic rock. The Jorge soils and Tahoma soils make up this unit with slight variations of very stony sandy loams and alluvial soils. The Tahoma soils described under the unit is typical of the Tahoma series with five to 15 percent of the surface area covered with cobblestones and boulders. The Jorge and Tahoma soils are moderately permeable

and well-drained. Slopes varies between three and four percent for the JwD soils and steeper, 15 to 30 percent, for the JwE soils. The average elevation of the watershed is 6,400 feet. The vegetation is semi-dense to dense stands of conifers, and cedar with an understory of mountain type shrubs. For these reasons, the entire project site was considered moderately permeable for soils hydrologic group B within the drainage flow calculations. According to the Soil Survey, the runoff is slow to medium with a slight erosion hazard.



0 1 2 3 4 5 10



CO310-FIG-1-1.DWG PLOT = AUG 27, 2004 SAVED AUG 18, 2004

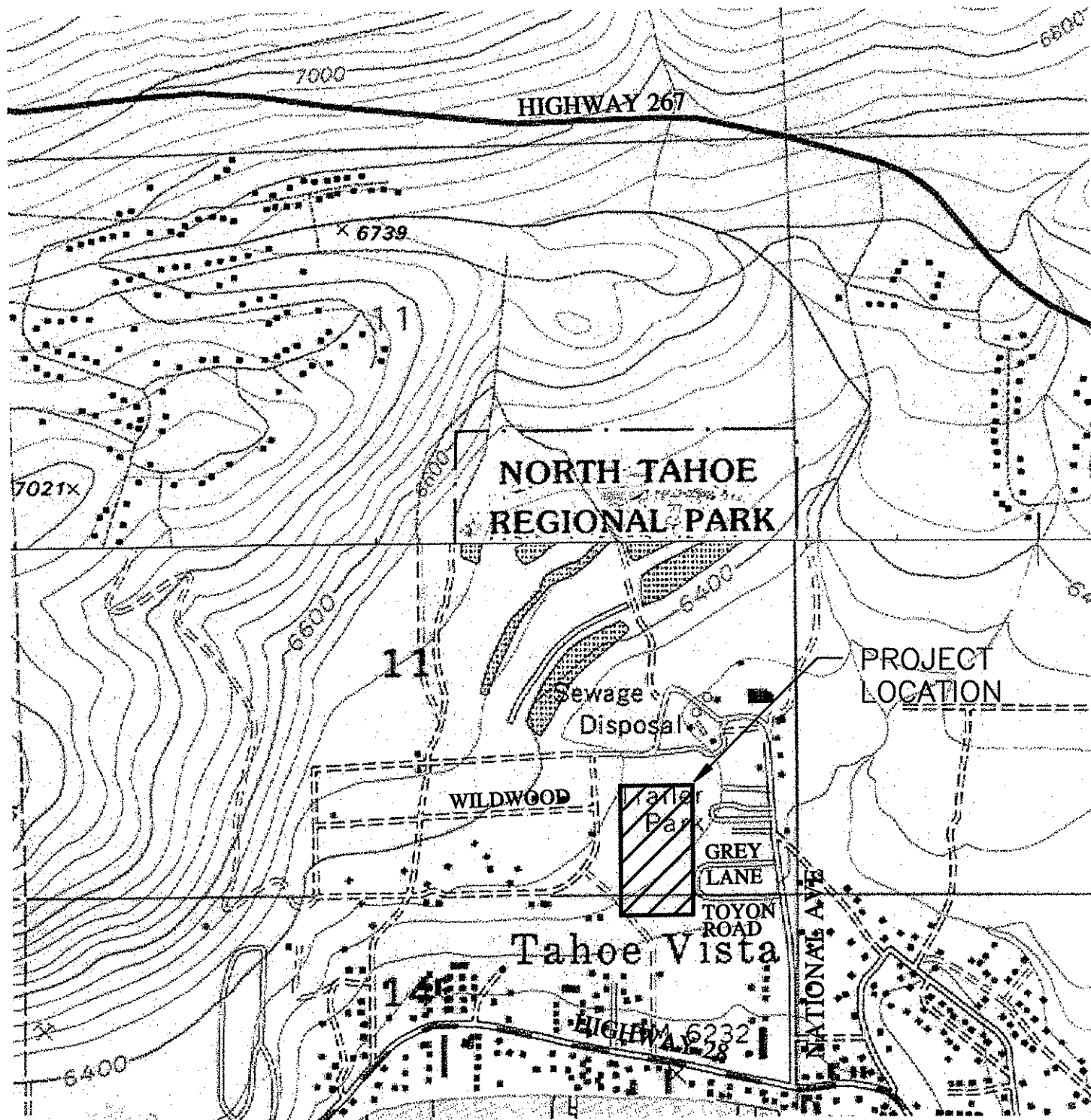


CEDAR GROVE APARTMENTS
AFFORDABLE HOUSING PROJECT-EIS/EIR
VICINITY MAP

FIGURE
1.1



SCALE: 1"=1000'



C0310-FIG-1-2.DWG PLOT = AUG 27, 2004 SAVED AUG 18, 2004



CEDAR GROVE APARTMENTS
AFFORDABLE HOUSING PROJECT-EIS/EIR
LOCATION MAP

FIGURE
1.2

LEGEND

----- OFFSITE RUNOFF
WATERSHED BOUNDARY



PROJECT AREA

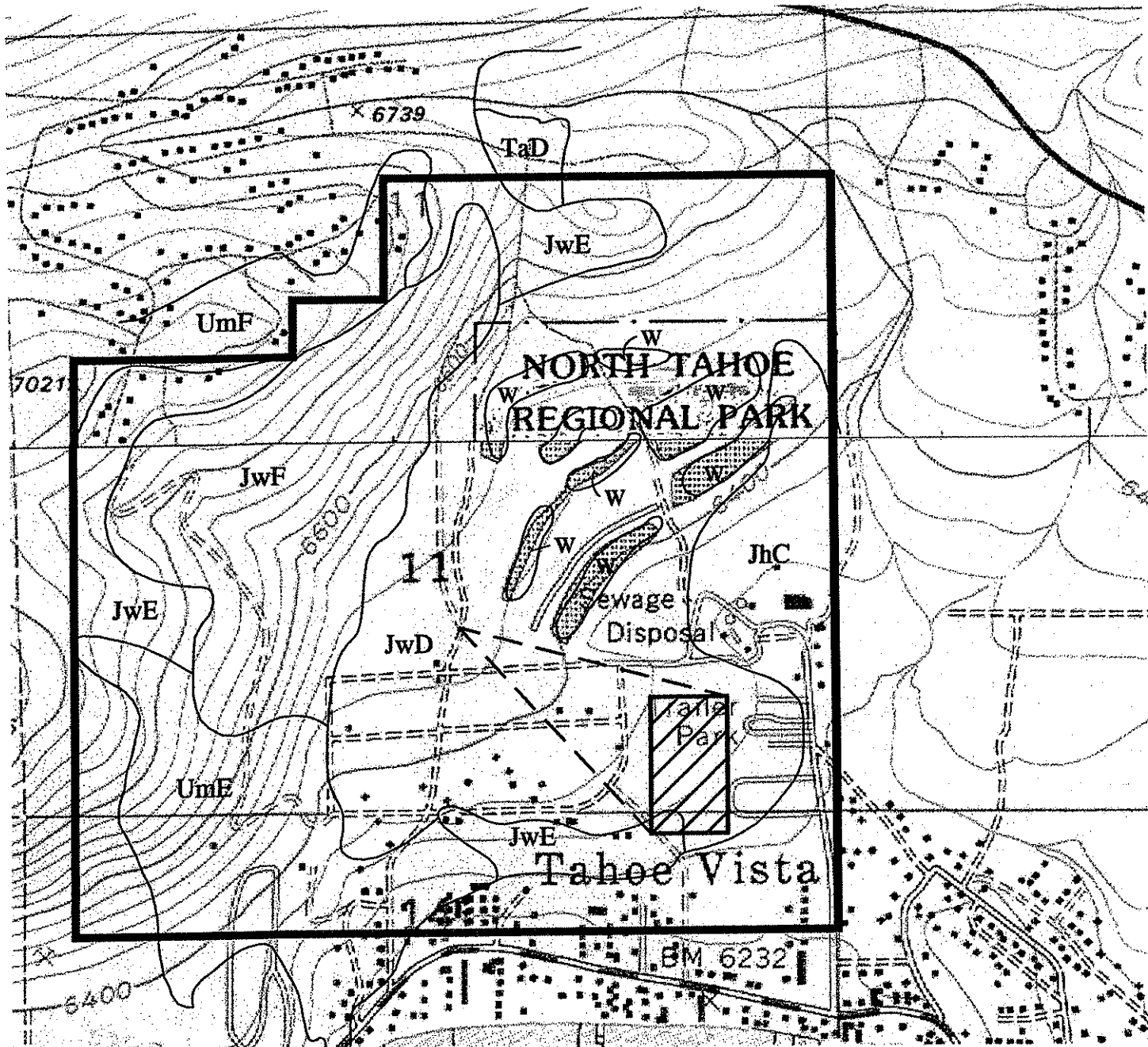
~~~~~ SOIL SURVEY  
SOIL TYPE

OFFSITE DRAINAGE AREA  
JwD = 15.35 Acres

PROJECT AREA = 12.5 Acres



SCALE: 1"=1000'



NOTE: SOIL TYPES FROM THE SOIL SURVEY,  
TAHOE BASIN AREA CALIFORNIA AND  
NEVADA, USDA, SCS AND FS, MARCH 1974

C0310-FIG-1-3.DWG PLOT = AUG 27, 2004 SAVED AUG 18, 2004



CEDAR GROVE APARTMENTS  
AFFORDABLE HOUSING PROJECT-EIS/EIR  
WATERSHED MAP

FIGURE  
1.3

#### ***1.4 Property Description***

The project will be constructed on a 12.5-acre parcel designated as APN 112-050-001. The property owner is the Mourelatos Family Limited Partnership, Idlewood Road, Tahoe Vista, California. The applicant is the Affordable Housing Development Corporation, Inc. (AHDC), whom has proposed the preferred alternative.

#### ***1.5 Project Description***

The proposed project consists of approximately 152 rental housing units within 23 buildings. An internal two-way looped roadway system includes access from Grey Lane, extending northwest and then south through the parcel, and then easterly with access to Toyon Road. Wildwood Road via Estates Drive near the northwest corner of the parcel is proposed to connect via a 12 foot roadway section for emergency use only.

Throughout the project, parking areas are provided along both sides of the roadway and within separate parking lots for the housing units. Stormwater runoff will be routed along curb and gutter with inlets conveying the standard 20-year, 1-hour volume to treatment facilities proposed along the roadway at feasible locations for water quality treatment and infiltration. The inlets and storm drain piping will route a portion of the runoff through a series of swales and ponds. The rest of the runoff will be routed along the roadway improvements.

The storm drain and drainage swales and the roadway together shall be designed to convey the 10-year peak flows as well as the 100-year peak flows. A Class 1 paved bike trail will skirt the eastern boundary of the parcel conveying bike traffic through the project area in a northerly direction to the North Tahoe Regional Park. The developer will be required to provide conveyance facilities for the 10- and 100-year storms per SWMM and install all water quality BMPs necessary to comply with stormwater runoff treatment requirements within the Tahoe Basin.

## **2. EXISTING AND PROPOSED DRAINAGE SYSTEMS**

### **2.1 *Off-Site Drainage***

Off-site drainage entering the project site is limited due to the small size of the watershed and the permeability of the landscaped and forested area of the Tahoe Estates Subdivision. Most of the upstream area runoff is diverted topographically and with improvements made to the North Tahoe Regional Park which directs drainage to the east and away from the project site near the north boundary of the project area. Figure 2.1 shows the approximately 15.5-acre watershed contributing flow to the project area.

### **2.2 *Existing On-Site Drainage***

The existing site is currently undeveloped and is heavily forested with minor open areas consisting of large quantities of mountain shrubs, building material debris, deadwood, pine needles and pinecones, and scattered boulders. There are numerous large trees over 24 inches diameter breast high (dbh) including several very large cedar trees and snags. The tree canopy is moderately dense to dense. There is no evidence of any drainage ways transecting the site. The site is very dry and no erosion, ditches, washes, channels, or streams were identified or verified upon the site visit performed for this preliminary analysis. All existing discharges from the site were determined to be from overland flow to the southern and southeastern boundaries.

### **2.3 *Proposed On-Site Drainage***

Conveyance facilities will be designed for the 10- and 100-year storms per SWMM. The 20-year, 1-hour roof runoff from all buildings, will be conveyed to standard dripline infiltration trenches or drywells that will be constructed adjacent to the buildings. Sidewalk runoff and bikeway runoff shall be directed to permeable areas of landscaping or to infiltration trenches where necessary. Figure 2.2 shows the remaining areas in which water quality BMPs and storm drainage improvements are planned for construction.





The roadway runoff must be first treated prior to infiltration with a treatment device. The type of treatment device shall be determined during design as is beyond the scope of the preliminary plan of development. The roadway was divided into similar runoff areas by evenly spaced inlets that convey the runoff to infiltration devices. For the proposed project there are approximately 19 locations that could be used for infiltration. Each runoff area is tied to a Point of Interest (POI), numbered 1 through 19 on Figure 2.2, and is associated with the hydrology and hydraulic calculations which are discussed further as a part of Section 3.

Treatment devices shall be constructed to treat the 20-year, 1-hour storm volume as required by the TRPA for removal of sediment and oils. Flows from larger events will be allowed to bypass the treatment basins and flow into the roadway drainage system. This system incorporates paved swales and curb and gutter drainage to the terminal discharge points at Grey Lane for the north

half of the project and at Toyon Road for the southern half of the project. In order to ensure that the storage system is available to treat and store runoff from a future storms, the infiltration systems will need to be drained over a 72-hour period. The SWMM requires that all storage facilities have a draw down within 72-hours. The time period also corresponds to the TRPA recommendations that a 34 - 72 hour draindown time should be incorporated into the design of all detention facilities in order to provide for vector control.

Overflow shall be incorporated to the infiltration basins and galleries in order for flows and runoff over the 20-year, 1-hour event volumes to be routed through the development and either exported by a regional system or by restricting the flow into these devices and diverting overflows and local runoff to the roadway. Runoff diverted to the roadway shall be directed to discharge points at Grey Lane and Toyon Road. Grey Lane and Toyon Road both terminate at National Avenue. Overland flow has historically been toward the south.

# LEGEND

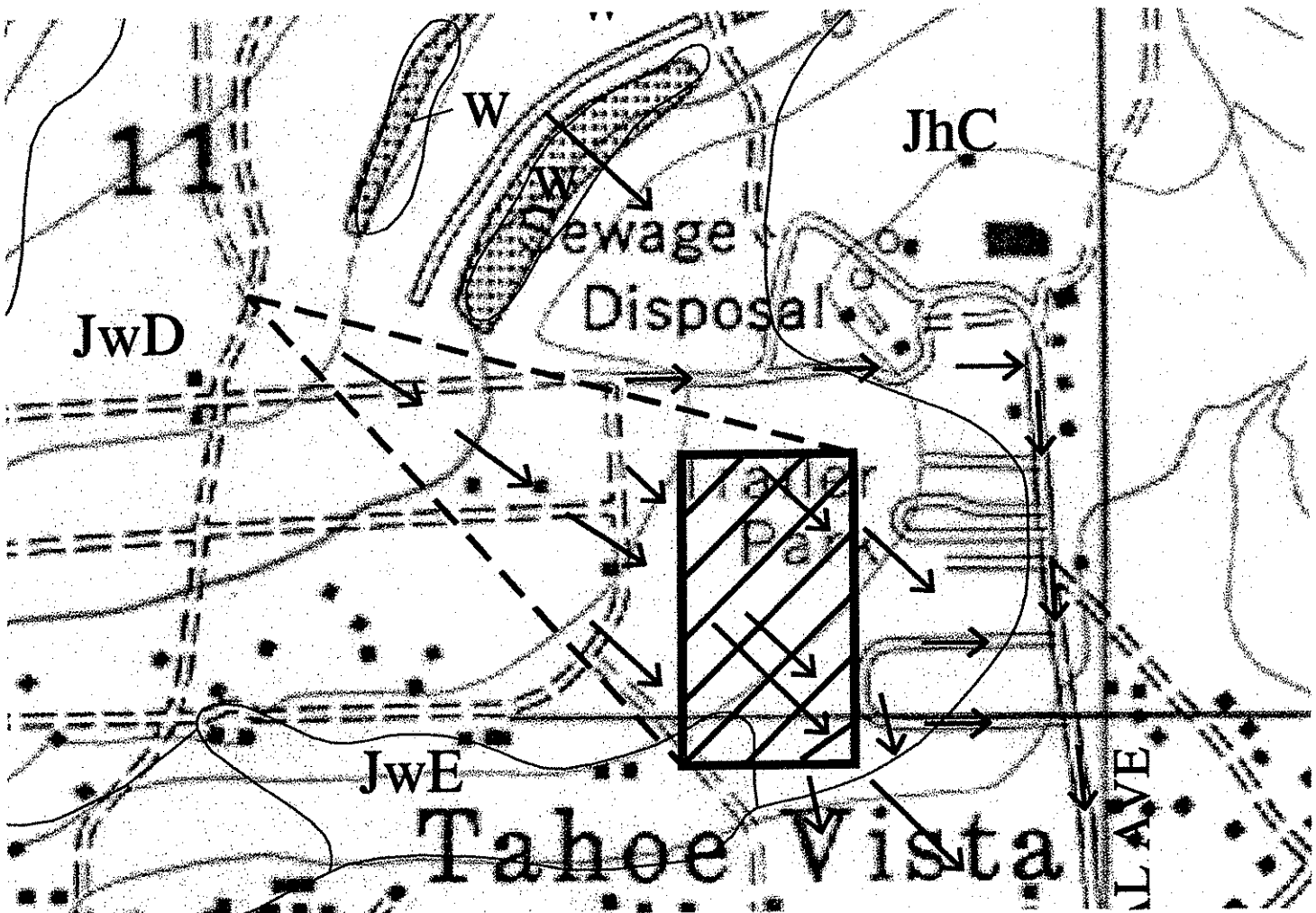
- OFFSITE RUNOFF WATERSHED BOUNDARY
-  PROJECT AREA
-  SOIL SURVEY SOIL TYPE
-  FLOW DIRECTION
-  100 YEAR FLOW PATH

OFFSITE DRAINAGE AREA  
JwD = 15.35 Acres

PROJECT AREA = 12.5 Acres



SCALE: 1"=500'



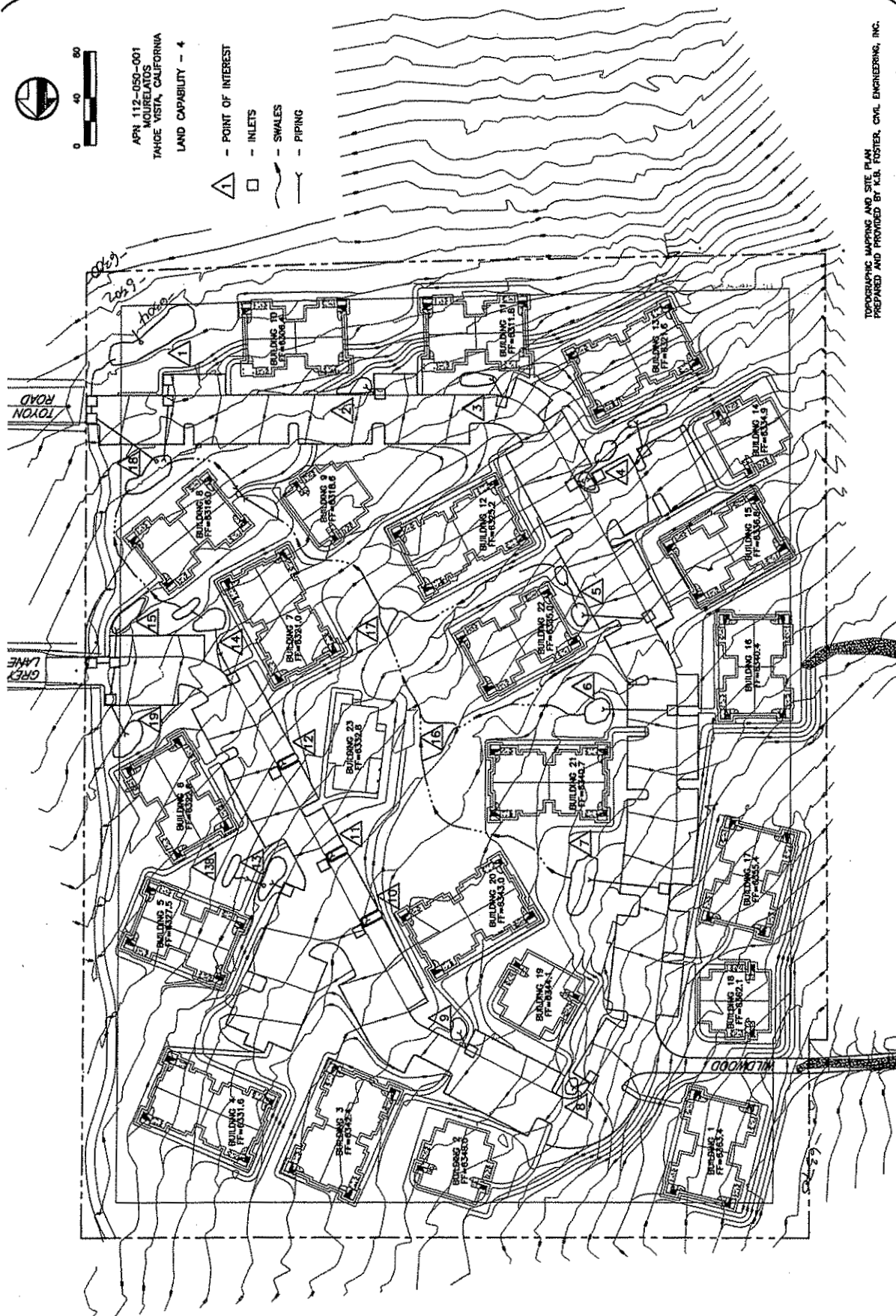
NOTE: SOIL TYPES FROM THE SOIL SURVEY,  
TAHOE BASIN AREA CALIFORNIA AND  
NEVADA, USDA, SCS AND FS, MARCH 1974

C0310-FIG-2-1.DWG PLOT = AUG 27, 2004 SAVED AUG 18, 2004



CEDAR GROVE APARTMENTS  
AFFORDABLE HOUSING PROJECT-EIS/EIR  
WATERSHED MAP

FIGURE  
2.1



TOPOGRAPHIC MAPPING AND SITE PLAN  
PREPARED AND PROVIDED BY K.B. FOSTER, CIVIL ENGINEERING, INC.  
CEN10-05-2-2.DWG PLOT = JUL 27, 2004 DWG DATE 15, 2004

## **2.4 Storm Drain System Design**

Figure 2.2, depicts a conceptual design of the storm drainage piping, consisting of primarily culverts, which possibly could be incorporated into the improvements planned for the Cedar Grove Apartments development. The pipe sizes have been established based upon the 10-year peak flow and slopes shown on the conceptual site plan. Detailed utility plans were not available as a part of this report. The hydraulic calculations prepared for this report (See Appendix) confirm that these sizes are suitable based upon the slopes and runoff flow rates calculated in the hydrology section. It should be noted that these sizes are not final and that adjustments may be made as a part of the final design, provided that the hydraulic calculations are updated to ensure that the required flows are being conveyed.

For the preferred alternative, a typical runoff area within the paved areas is approximately 5000 to 7200 square feet in size and generates peak flows of 0.5 cfs per acre, for a 10-year storm event. The slope across the road is approximately 1.3 percent and the range in culvert pipe sizing is 12 to 15 inches in diameter for concrete pipe. Culverts conveying peak flows to the two major downstream points of interest should be able to convey the 10-year peak flows at those points, POI - 1 and 14, and are 13.5 cfs, and 6.1 cfs, respectively. These culvert sizes should be in the range of 18 to 24 inches.

The final design shall incorporate the conveyance of the pre-project 100-year event through the site and the bypass of the culvert piping and roadway grades to prevent damage to property.

### 3. HYDROLOGY/HYDRAULICS EVALUATION

#### 3.1 General

This portion of the drainage study describes the methodology utilized for the development of the preliminary hydrology and hydraulics for the proposed Cedar Grove Apartments EIS/EIR, and the resulting conclusions.

The Stormwater Management Manual (SWMM) prepared by the Placer County Flood Control and Water Conservation District is the basis for the project requirements for hydrology, conveyance and analysis of downstream impacts. The SWMM dictates that proposed development shall not adversely impact upstream or downstream drainage facilities, which generally requires detention of the excess runoff volume generated by the proposed project conditions. The Tahoe Regional Planning Agency (TRPA) and the California Regional Water Quality Control Board, Lahontan Region (Lahontan) also require detention of runoff for water quality treatment purposes. These volume requirements are considered independently of each other for the purposes of this report. The design storm parameters for this report are outlined in the following table:

**Table 3.1**  
**Design Storm Requirements**

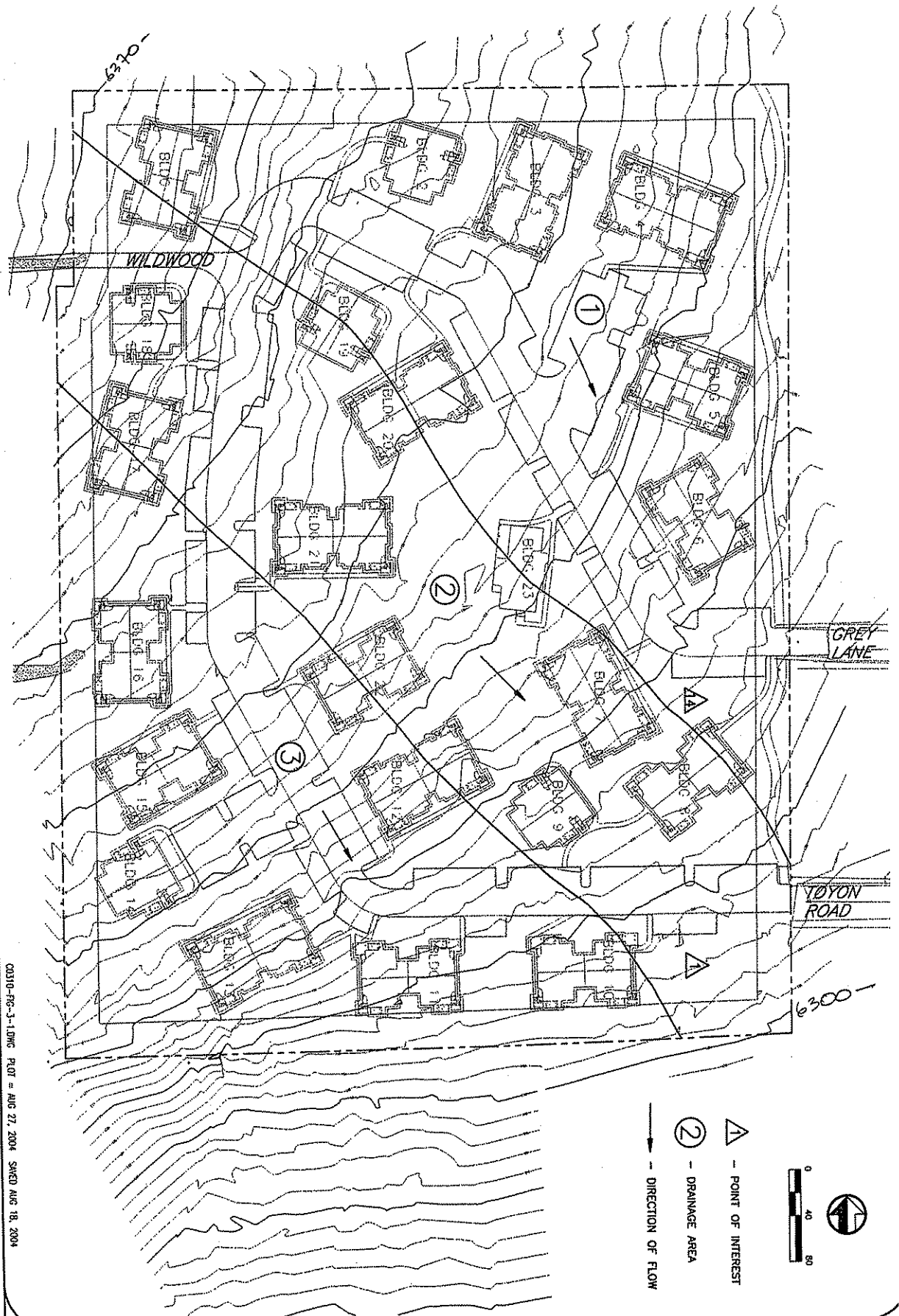
| Design Element                       | Storm Event       | Source             |
|--------------------------------------|-------------------|--------------------|
| Conveyance (Flow Rate)               | 10-Year, 24-Hour  | Placer County SWMM |
|                                      | 100-Year, 24-Hour | Placer County SWMM |
| Storage (Flow Volume*)               | 10-Year, 24-Hour  | Placer County SWMM |
|                                      | 100-Year, 24-Hour | Placer County SWMM |
| Storage/Infiltration (Water Quality) | 20-Year, 1-Hour   | TRPA/Lahontan      |

\* Difference between pre- and post-project hydrographs for these storms.

#### 3.2 Hydrology

The watershed area is approximately 28 acres; therefore, the hydrology of the project watershed has been evaluated using the methodology for small watersheds described in the SWMM.

The project watershed is divided into eight subwatersheds to allow for evaluation of the existing and proposed drainage conditions. Figures 3.1 and 3.2 show the subwatersheds for existing and proposed conditions, respectively.



0310-Fig. 3-1.DWG. PLOT = AUG 27, 2004. SAVED AUG 18, 2004



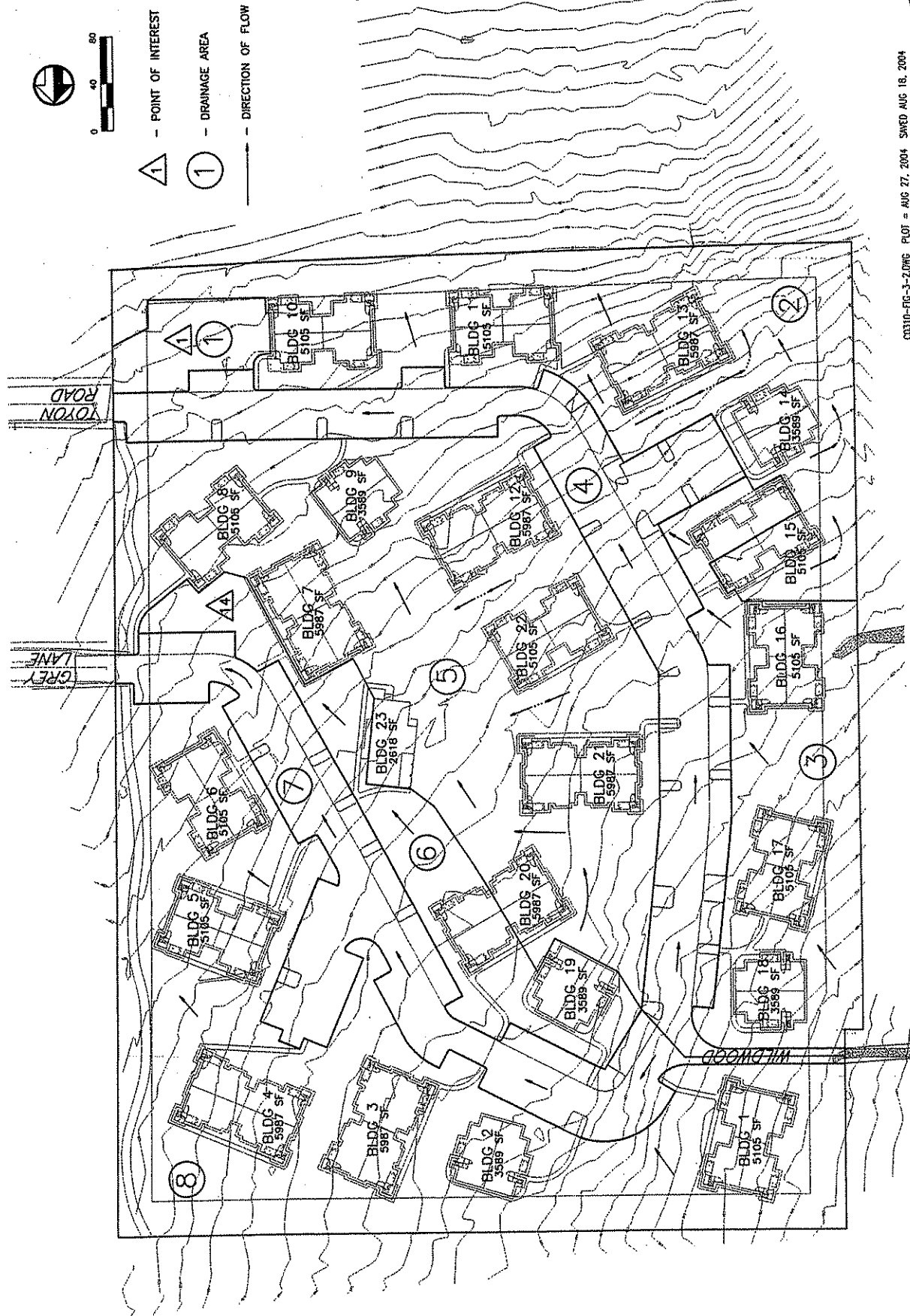
CEDAR GROVE APARTMENTS  
AFFORDABLE HOUSING PROJECT-EIS/EIR  
EXISTING DRAINAGE AREAS

FIGURE  
3.1



CEDAR GROVE APARTMENTS  
AFFORDABLE HOUSING PROJECT-EIS/EIR  
PROPOSED DRAINAGE AREAS

FIGURE  
3.2



C0310-FIC-3-2.DWG PLOT = AUG 27, 2004 SAVED AUG 18, 2004

The delineation of the existing sub-watersheds is based on topography prepared for the Cedar Grove Apartments Proposed Project by K. B. Foster Civil Engineering, Inc. Delineation of the proposed subwatersheds is based on the conceptual improvements described within the Notice of Preparation by the TRPA, and on the mapping provided by K. B. Foster for the proposed project. All other alternatives are assumed to be of lesser impact; therefore, the calculations made within this drainage report would be conservative and subject to adjustment during the design phase of the project. The K. B. Foster topographic mapping was also utilized to determine the building areas, and to estimate the total impervious area percentage for each watershed for the existing and proposed project conditions. Area calculations are located in the Appendix.

The SWMM methodology determines the peak flow from a watershed based on the watershed's area and surface characteristics, as well as its response time. The response time, for the purposes of this study, is divided into two components, overland flow and collector, or channelized, flow. Based on the existing topographic mapping and assumptions regarding the proposed improvements as described previously, a flow path is designated for the overland flow from the offsite area and the longest flow path along the roadway. The overland flow response time is calculated based on the overland flow length and slopes, from existing and proposed mapping, and the roughness coefficient of the surface.

The collector flow response time is based on the size of the contributing area, and the roughness, length, slope, and geometry of the conveyance system. As with the overland flow parameters, the length and slope are determined from available mapping, with the roughness coefficient and geometry are based on the assumption of a typical concrete curb and gutter section for the post-development condition. The contributing area is set equal to the area of the subwatershed in question. The sum of the overland and collector response time components represents the response time for the subwatershed and are included in the Appendix, for both the pre-project and post-project conditions.

The response time, in combination with project area's elevation, is then used to determine the unit peak discharge for each subwatershed from Figures 5-3A (10-Year Storm) and 5-3C (100-Year Storm) in the SWMM. The unit peak discharges obtained from the SWMM are based on a response time of 60 minutes since the calculated time of response for the project area exceeds 60 minutes.

The peak flow from each subwatershed and point of interest is calculated from the unit peak discharge for each storm, and the characteristics of each watershed. The peak flow is the product of the unit peak discharge and the watershed area, with reductions due to infiltration of runoff through pervious surfaces in the watershed. Pervious surfaces are assigned a general infiltration rate based on factors such as land use, hydrologic condition of the ground cover, and hydrologic soil group for the native soils. For the purposes of this study, the general infiltration rates of 0.20 to 0.26 inches per hour is used for pervious areas within the project watershed. This rate is obtained from Table 5-3 in the SWMM and is 0.20 for an area in good condition, where native soils are categorized under hydrologic soil group B. The general infiltration rate is further

reduced in the proposed condition by applying a snow coverage factor per SWMM Table 5-4 for an elevation of 6,400 feet east of the Sierra crest. This table indicates that approximately 88% of the watershed is snow-covered and should be considered impervious in runoff calculations. The peak flow for both the 10-year and 100-year events is calculated for both the existing and proposed conditions, over the entire site. The following table summarizes the peak flows for the existing and proposed conditions for the 10-year and 100-year storms.

**Table 3.2**  
**Summary of Peak Flows**

| Point of Interest | Tributary Watersheds | Area (AC) | Percent Pervious | Response Time (min) | 10-Year Peak Flow (cfs) | 25-Year Peak Flow (cfs) | 100-Year Peak Flow (cfs) |
|-------------------|----------------------|-----------|------------------|---------------------|-------------------------|-------------------------|--------------------------|
| Pre-development   |                      |           |                  |                     |                         |                         |                          |
| POI - 1           | 1, 2, Offsite        | 23.9      | 50 - 100         | 105                 | 11.4                    | 15.0                    | 18.6                     |
| POI -14           | 1, ½ Offsite         | 12.4      | 50 - 100         | 105                 | 5.9                     | 7.8                     | 9.6                      |
| Post-development  |                      |           |                  |                     |                         |                         |                          |
| POI - 1           | 1, 3-8               | 26.0      | 50               | 75                  | 12.6                    | 16.5                    | 20.4                     |
| POI - 14          | 6, 7, 8              | 12.5      | 50               | 75                  | 6.0                     | 7.9                     | 9.8                      |

The pre- and post-project flows are compared to determine whether or not the development would have an adverse impact on downstream facilities. Any adverse impact would need to be mitigated through improvement of the downstream facilities, or by limiting the discharges from the project site to no more than the existing condition for each design storm. The calculations indicate that, based only on Placer County criteria, there is no significant adverse impact from the proposed development. The detention facilities shall discharge at a pre-development rate and, therefore, no additional mitigation is required. In addition, storage and infiltration requirements for TRPA and Lahontan would further reduce the discharge from the proposed project.

In order to mitigate the affects of the development, the project may include design of up to two local detention basins. The storage capacity and outlet design of each shall be designed to limit outflows to the pre-development outflow hydrographs for the design storm events per the SWMM. Required capacity is a function of objective outflows, design inflows, and required freeboard. Design decisions should be coordinated with the Flood Control District, including the methods to be used and the use of approved inflow hydrographs.

In order to provide an estimate of storage volume in the downstream basin or point of interest, preliminary detention calculations were made based on a simplified hydrograph procedure. Volume estimates were generated by using the pre-development inflow rates, objective peak

outflow rates (discounted for uncertainty), and by utilizing the calculated peak response times per the SWMM. The outlets to the detention basin shall be designed to provide for the 2-, 10-, 25-, and 100-year objective outflows. Spillways shall be provided and shall decrease flows by providing a surcharge storage capacity, and all basins shall be designed with the required freeboard.

The estimated base volumes necessary to detain the 10-, 25-, and 100-year storm are shown in the table below. The volumes do not account for surcharge storage, freeboard, or losses due to storage elsewhere in the system.

**Table 3.3**  
**Basin Storage Volumes**

| Storm Frequency | Pre-development Inflow (cfs) | Objective Outflow (cfs) | Post-development $\Delta Q_p$ (cfs) | Time, Base (min) | Time, Base (sec) | Required Storage Volume (cf) |
|-----------------|------------------------------|-------------------------|-------------------------------------|------------------|------------------|------------------------------|
| 10              | 11.4                         | 11.3                    | 1.2                                 | 340              | 20,400           | 13,260                       |
| 25              | 15.0                         | 14.8                    | 1.5                                 | 335              | 20,079           | 17,067                       |
| 100             | 18.6                         | 18.4                    | 1.8                                 | 425              | 25,500           | 25,500                       |

Detention basin sizing can be determined utilizing soils data, groundwater depth information and design guidelines provided in the SWMM for local basins. In the vicinity of POI-1, an area of approximately 2,400 sf is available to provide for a detention facility. Storage depth would need to be 10.6 feet to accommodate the 100-year event and attenuate the flows from all runoff. To accommodate the 25-year event, a base depth of 7.1 feet would be necessary.

If two basins were designed, the sizes of the basins and depths could be modified. In the area of POI-14 and 15 there is a 1,800 sf area available. If one-half the flows were routed to this detention area, then the base depths of the basins would be reduced accordingly. The following Table 3.4 gives approximate sizing of the basins.

**Table 3.4**  
**Preliminary Basin Sizing**

| Frequency | One Basin | Two Basins |        |
|-----------|-----------|------------|--------|
|           | POI-1     | POI-1      | POI-14 |
| 10 Years  | 5.5       | 2.8        | 3.7    |
| 25 Years  | 7.1       | 3.6        | 4.7    |
| 100 Years | 10.6      | 5.3        | 7.1    |

Other areas that could be incorporated into a plan for detention of flows are POI - 16 through POI - 19. Figure 2.2 may be used to reference possible basin locations.

During design routing of the flow and the determination of discharge rates and locations should be coordinated with the Placer County Flood Control District. The SWMM guidelines should be used to determine allowable depths, freeboard and associated maintenance requirements for design of the detention facilities.

The basin discharge from the outlet or spillway would be directed to a smaller area than that occurring naturally. Higher velocities would need to be mitigated with energy dissipaters to prevent erosion downstream. Development of conveyance system downstream of the detention facilities should be analyzed during design.

### **3.3    *Hydraulics***

Due to the preliminary nature of the proposed project design, the hydraulics of the proposed main storm drain system have been calculated using open channel flow methods only. Detailed hydraulic calculations are included in the Appendix. These calculations indicate that the capacity of the proposed storm drain is more than adequate to convey even the 100-year flows generated by the proposed development conditions and existing 100-year flow from the upstream, off-site watershed. The proposed storm drain currently incorporates concrete pipe roughly coinciding with the grades of the existing terrain in the area. When a more detailed design of the storm drain system is justified, the hydraulics of the closed system will be evaluated, and pipe invert and structure rim elevations set. If necessary, alternate pipe materials can be utilized if required for enhanced conveyance capacity.

Due to the fact that the proposed development will not significantly increase the runoff beyond the existing conditions, and will discharge at a rate equal to the pre-development rate, for the design storm, no hydraulic evaluation has been performed for downstream facilities.

\\Lt-server\c-drive\02\CJOBS\03JOBS\C0310-Cedar Grove\C0310 draft drainage report.wpd

## **APPENDIX**

**APPENDIX A**  
**HYDROLOGY CALCULATIONS**



Job #: C0310 Page: 1 of 8

Job Title: Cedar Grove Apt/EIS/ER Date: 8-25-04

Engineer: \_\_\_\_\_

JWA CONSULTING ENGINEERS, INC.

Preliminary Design Calculations - Overall Hydrology  
based on the Placer County, Flood Control and Water Conservation  
District Stormwater Management Manual, Feb. 1994 and  
Addendum I, Oct. 1997. (SWMM)

Undeveloped Condition - Assume entire site drains to POI-1,  
SE CORNER

Calculate  $t_r$ , response time

$L_1 = 500'$  Overland Flow

$L_2 = 1150'$  overland flow, unconnected flow, and  
landscaped areas throughout Subdivision.

$L_3 = 600'$  overland flow (project area.)  
no identified channels  
may have areas of shallow concentrated  
flow, unidentified

| Path  | Distance         | slope | Roughness                                         | Cover                    | Soil Type |
|-------|------------------|-------|---------------------------------------------------|--------------------------|-----------|
| L1    | 500              | .040  | 0.40                                              | Open Brush               | JWD       |
| L2    | 1150             | .036  | 0.50                                              | Woodland<br>Canopy < 50% | JWD       |
| L3    | 600              | .036  | 0.80                                              | Woodland<br>Canopy > 50% |           |
| Total | 2250             |       |                                                   |                          |           |
| Path  | Hydrologic Group |       | Constant Infiltration<br>Rates, I Table 5-3, SWMM |                          |           |

L1 B 0.20

L2 B (unconnected) 0.24

L3 B 0.26

$I_{weighted} = \frac{(1000) + 276 + 156}{2250} = \underline{\underline{0.24 \text{ inches/hr}}}$



Job #: CD310

Page: 2 of 8

Job Title: Cedar Grove Apt/Eskic Date: 8-25-04

Engineer: \_\_\_\_\_

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Overland flow response time - Undeveloped Condition

$$t_r = 0.355 (nL)^{0.6} \quad (\text{Eq. 5-3, SWMM})$$

$$t_{r1} = \frac{0.355 (0.40 \times 1500)^{0.6}}{(0.036)^{0.3}} = 22.39$$

$$t_{r2} = \frac{0.355 (0.50 \times 1150)^{0.6}}{(0.036)^{0.3}} = 43.57$$

$$t_{r3} = \frac{0.355 (0.80 \times 600)^{0.6}}{(0.036)^{0.3}} = 39.09$$

$$t_{r\text{TOT}} = 105 \text{ min}$$

Unit Peak Flows

10yr unit peak flow Fig 5-3A

25yr unit peak flow Fig 5-3B

100yr unit peak flow Fig 5-3C

(2yr not provided in the SWMM, flows calculated assumed greater.)

Average elevation 6420 - 6360 - 6390, use 6400'

From Figures,

For  $t_r$  or greater:  $q_{100} = 0.8 \text{ cfs/acre}$  $q_{25} = 0.65 \text{ cfs/acre}$  $q_{10} = 0.50 \text{ cfs/acre}$ Infiltration Factor - Undeveloped

$$F_i = I \left( 1 + \frac{1}{(1.3 + 0.0005E)} \right)$$

$$F_i = 0.24 \left( 1 + \frac{1}{(1.3 + 0.0005(6400))} \right)$$

$$F_i = 0.29 \text{ cfs/acre}$$



Job #: 00310

Page: 3 of 8

Job Title: Cedar Grove Apts BS/EIR

Date: 8-25-04

Engineer: \_\_\_\_\_

JWA CONSULTING ENGINEERS, INC.

Adjustment to Peak Flows for Infiltration Eqn 5-6

$$Q_p = qA - A_p F_i$$

Area, pervious

$$Q_{100} = 0.8(12.5 + 15.53) - [.12(12.5) + (.12)(.50)(15.53)](.29)$$

$$Q_{100} = 22.42 \text{ cfs} - 0.70 \text{ cfs} = 21.7 \text{ cfs}$$

$$Q_{75} = 0.65(12.5 + 15.53) - 0.70 = 17.5 \text{ cfs}$$

$$Q_{50} = 0.50(12.5 + 15.53) - 0.70 = 13.3 \text{ cfs}$$

Assume 50% of offsite area is impervious and  
88% of all area are considered impervious due to  
snow cover (Table 5-4)

Developed Condition

Assume Entire site drains to  
POT-1, SE Corner

Calculate  $t_r$ 

| Path           | Distance | Slope | Roughness | Cover        | Soil Type |
|----------------|----------|-------|-----------|--------------|-----------|
| L <sub>1</sub> | 500      | .040  | 0.40      | Open Brush   | TWD       |
| L <sub>2</sub> | 1150     | .036  | 0.50      | Woodland/SED | TWD       |
| L <sub>3</sub> | 600      | .036  | 0.30      | Woodland/Res | TWD       |

| Path           | Hydrologic Group  | Constant Infiltration Rate, I |
|----------------|-------------------|-------------------------------|
| L <sub>1</sub> | B                 | 0.20                          |
| L <sub>2</sub> | B Res. Landscaped | 0.24                          |
| L <sub>3</sub> | B Not Landscaped  | 0.25                          |

$$\text{Weighted I} = \frac{100 + 276 + 150}{2250} = 0.23 \text{ inches/hr.}$$



Job #: C0310

Page: 4 of 8

Job Title: Cedar Grove Apts EIS/ER

Date: 8-25-04

Engineer: \_\_\_\_\_

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Developed Condition  
Overland Flow Response Time, (Remains same for off site area)

$$t_{L1} = 22.39$$

$$t_{L2} = 43.57$$

Assume channelized runoff across site is roadway curb & gutter flow

Curb and gutter flow: open channel

Assume 2 fps,  $S = .036$ ,  $L = 1080$  LF

$$t_{L3} = 9.0 \text{ minutes}$$

If  $S = 5\%$  on Roadway,  
velocities may be higher.

$$\text{Total } t_r = 75 \text{ minutes}$$

(Plan master calc shows 7 fps,  
translates to 3 minutes, leaves  
 $t_{r \text{ total}} \approx 69.0$  minutes)

Unit peak flows can be derived from Figures 5-3 A, B, C,  
Due to  $t_r > 60$  minutes, the resulting unit peak flows  
are the same for the Developed Condition

$$q_{100} = 0.8 \text{ cfs/ac}$$

$$q_{25} = 0.65 \text{ cfs/ac}$$

$$q_{10} = 0.50 \text{ cfs/ac}$$

Infiltration Factor - Developed Condition

$$F_i = 0.23 (1 + 1 / (1.3 + 0.0005 (6400)))$$

$$= 0.28 \text{ cfs/ac}$$

Job #: CO310Page: 5 of 8Job Title: Cedar Grove Apts EIS/ERDate: 8-25-04

Engineer: \_\_\_\_\_

JWA CONSULTING ENGINEERS, INC.

Adjustment for Infiltration Developed Conditions

$$Q_p = q_A - A_p F_i \quad (\text{Eqn 5-6, Table 5-4})$$

$$Q_{100} = 0.8 (12.5 + 15.53) - [12.50(50) + 0.50(15.53)] (0.12)(0.28) \quad \leftarrow \text{snow adj.}$$
$$= 22.43 - 0.47 = 22.0 \text{ cfs}$$

$$Q_{25} = 0.65 (12.5 + 15.53) - 0.47 = 17.8 \text{ cfs}$$

$$Q_{10} = 0.50 (12.5 + 15.53) - 0.47 = 13.5 \text{ cfs}$$

Assume 50% impervious offsite and onsite, 88% imp due to snow

Summary Table

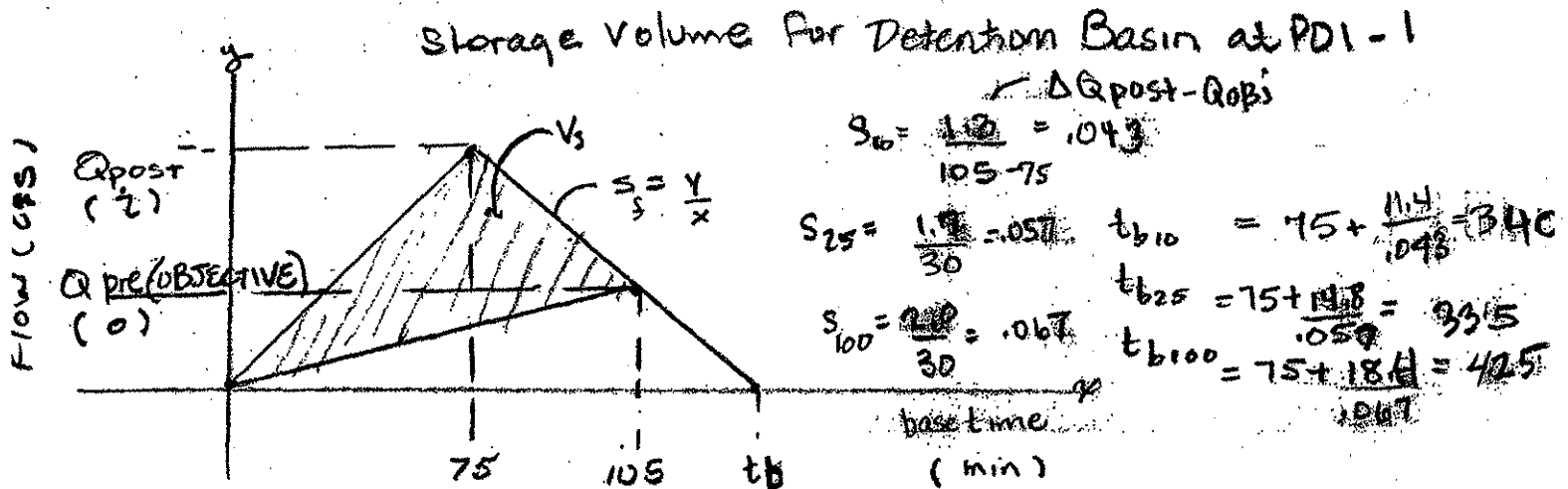
Assumes all flow is directed to Point A (POI 1)

| Flow      | PREDEVELOPED | POST DEVELOPED | NET INCREASE % |
|-----------|--------------|----------------|----------------|
| $Q_{100}$ |              |                |                |
| 100       | 21.7 cfs     | 22.0 cfs       | 1.4%           |
| 25        | 17.5 cfs     | 17.8 cfs       | 1.7%           |
| 10        | 13.3 cfs     | 13.5 cfs       | 1.5%           |

Adjustment of Areas  
Area 3 existing flows offsite  
Area 2 proposed flows offsite

Table 3.2  
Summary of Peak Flows

| POINT OF INTEREST | TRIB. AREA     | AREA | % PERVIOUS | Response Time | 10 YR Peak cfs | 25 YR Peak cfs | 100 YR Peak cfs |
|-------------------|----------------|------|------------|---------------|----------------|----------------|-----------------|
| Predevelopment    |                |      |            |               |                |                |                 |
| POI-1             | 1, 2, OFFSITE  | 23.9 | 50-100     | 105           | 11.4           | 15.0           | 18.6            |
| POI-14            | 1, 1/2 OFFSITE | 12.4 | 50-100     | 105           | 5.9            | 7.8            | 9.6             |
| POI-1             | 1, 3-8         | 26.0 | 50         | 75            | 12.6           | 16.5           | 20.4            |
| POI-14            | 6, 7, 8        | 12.5 | 50         | 75            | 6.0            | 7.9            | 9.8             |



$$V_s = 0.5 t_b \frac{(Q_i - Q_o)}{\text{sec}} \quad \begin{matrix} Q_i = \text{peak inflow} \\ Q_o = \text{outflow (objective)} \end{matrix}$$

$$V_{s10} = 0.5 (20,400) (1.3) = 13,260 \text{ cf}$$

$$V_{s25} = 0.5 (20,079) (1.7) = 17,067 \text{ cf}$$

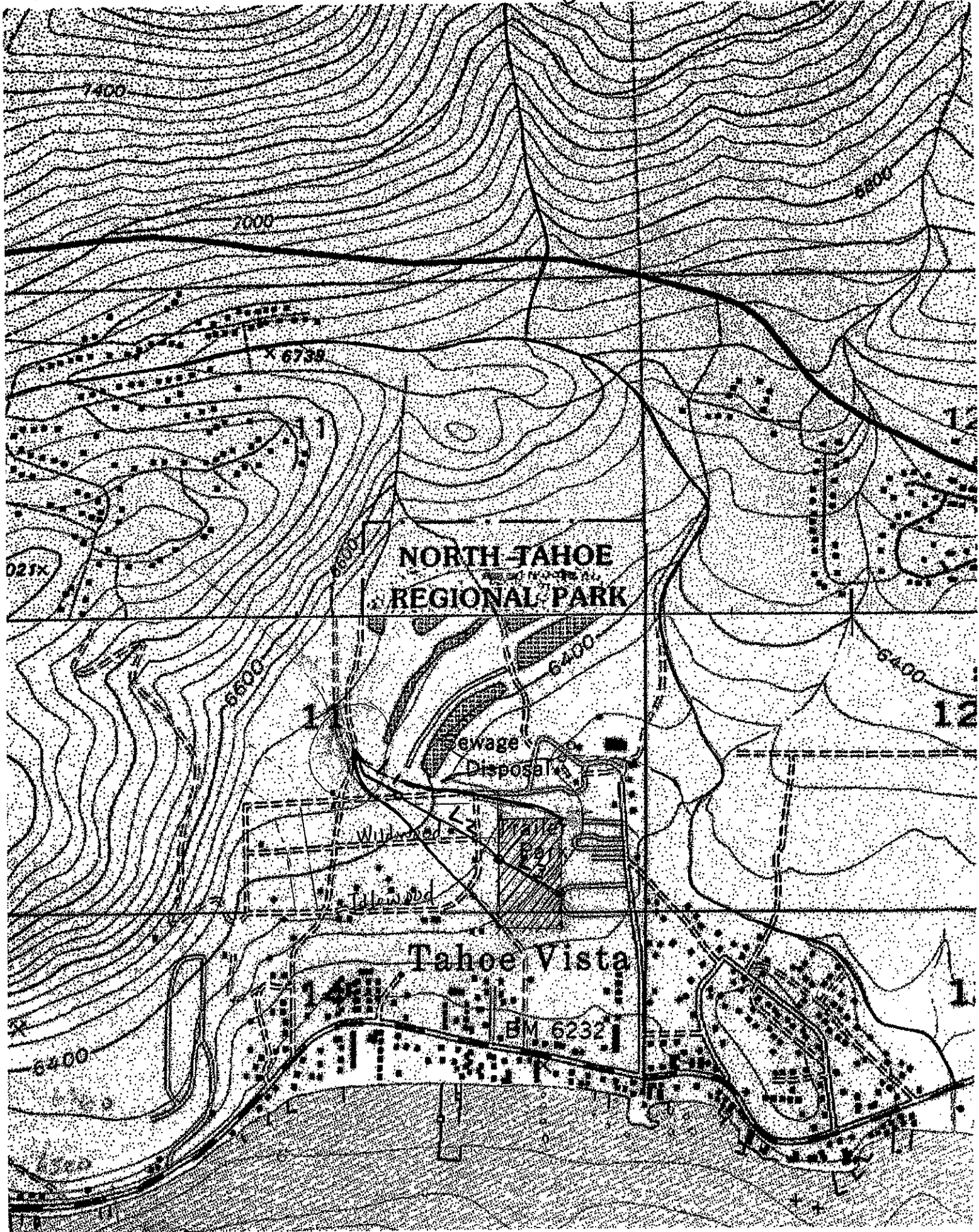
$$V_{s100} = 0.5 (25,500) (2.0) = 25,500 \text{ cf}$$

Cedar Grove Apts EIS/EIR 8-25-04

Engineer \_\_\_\_\_

Table 3.3  
Storage Volumes

| storm freq | PreDev<br>inflow<br>cfs | OBJ<br>OUTFLOW<br>cfs | Post-Ave<br>ΔQp<br>cfs | t <sub>b</sub><br>min. | t <sub>b</sub><br>sec | V <sub>s</sub><br>cfs |
|------------|-------------------------|-----------------------|------------------------|------------------------|-----------------------|-----------------------|
| 10         | 11.4                    | 11.3                  | 1.2                    | 340                    | 20,400                | 13,26<br>13,260       |
| 25         | 15.0                    | 14.8                  | 1.5                    | 335                    | 20,079                | 17,06<br>17,067       |
| 100        | 18.6                    | 18.4                  | 1.8                    | 425                    | 25,500                | 25,56<br>25,500       |



**Cedar Grove Apartments EIS/EIR  
Proposed Project Drainage Areas**

| Area ID | Area (SF) |
|---------|-----------|
| 1       | 8,282     |
| 2       | 85,561    |
| 3       | 51,776    |
| 4       | 57,277    |
| 5       | 122,938   |
| 6       | 30,945    |
| 7       | 45,185    |
| 8       | 130,976   |
|         | 532,940   |

**Existing Drainage Areas**

| Area ID | Area (SF) | Area (Ac) |
|---------|-----------|-----------|
| 1       | 202,271   | 4.6       |
| 2       | 162,038   | 3.7       |
| 3       | 168,620   | 3.9       |

**APPENDIX B**  
**HYDRAULIC CALCULATIONS**

# Worksheet Worksheet for Circular Channel

| Project Description |                               |
|---------------------|-------------------------------|
| Worksheet           | Circular Channel - paved area |
| Flow Element        | Circular Channel              |
| Method              | Manning's Formula             |
| Solve For           | Channel Depth                 |

| Input Data           |                |
|----------------------|----------------|
| Mannings Coefficient | 0.013          |
| Slope                | 0.013000 ft/ft |
| Diameter             | 15 in          |
| Discharge            | 0.08 cfs       |

| Results          |                        |
|------------------|------------------------|
| Depth            | 0.09 ft                |
| Flow Area        | 4.0e-2 ft <sup>2</sup> |
| Wetted Perimeter | 0.69 ft                |
| Top Width        | 0.65 ft                |
| Critical Depth   | 0.11 ft                |
| Percent Full     | 7.3 %                  |
| Critical Slope   | 0.006334 ft/ft         |
| Velocity         | 1.98 ft/s              |
| Velocity Head    | 0.06 ft                |
| Specific Energy  | 0.15 ft                |
| Froude Number    | 1.40                   |
| Maximum Dischar  | 7.92 cfs               |
| Discharge Full   | 7.36 cfs               |
| Slope Full       | 0.000002 ft/ft         |
| Flow Type        | Supercritical          |

# Worksheet Worksheet for Circular Channel

| Project Description |                      |
|---------------------|----------------------|
| Worksheet           | Circular Channel - 1 |
| Flow Element        | Circular Channel     |
| Method              | Manning's Formula    |
| Solve For           | Channel Depth        |

| Input Data           |                |
|----------------------|----------------|
| Mannings Coefficient | 0.013          |
| Slope                | 0.013000 ft/ft |
| Diameter             | 24 in          |
| Discharge            | 13.40 cfs      |

| Results          |                     |
|------------------|---------------------|
| Depth            | 1.02 ft             |
| Flow Area        | 1.6 ft <sup>2</sup> |
| Wetted Perimeter | 3.19 ft             |
| Top Width        | 2.00 ft             |
| Critical Depth   | 1.32 ft             |
| Percent Full     | 51.1 %              |
| Critical Slope   | 0.005899 ft/ft      |
| Velocity         | 8.29 ft/s           |
| Velocity Head    | 1.07 ft             |
| Specific Energy  | 2.09 ft             |
| Froude Number    | 1.63                |
| Maximum Dischar  | 27.74 cfs           |
| Discharge Full   | 25.79 cfs           |
| Slope Full       | 0.003509 ft/ft      |
| Flow Type        | Supercritical       |

# Worksheet Worksheet for Circular Channel

| Project Description |                               |
|---------------------|-------------------------------|
| Worksheet           | Circular Channel - culvert 14 |
| Flow Element        | Circular Channel              |
| Method              | Manning's Formula             |
| Solve For           | Channel Depth                 |

| Input Data           |                |
|----------------------|----------------|
| Mannings Coefficient | 0.013          |
| Slope                | 0.013000 ft/ft |
| Diameter             | 24 in          |
| Discharge            | 6.00 cfs       |

| Results          |                |
|------------------|----------------|
| Depth            | 0.66 ft        |
| Flow Area        | 0.9 ft²        |
| Wetted Perimeter | 2.44 ft        |
| Top Width        | 1.88 ft        |
| Critical Depth   | 0.87 ft        |
| Percent Full     | 32.8 %         |
| Critical Slope   | 0.004645 ft/ft |
| Velocity         | 6.69 ft/s      |
| Velocity Head    | 0.70 ft        |
| Specific Energy  | 1.35 ft        |
| Froude Number    | 1.71           |
| Maximum Dischar  | 27.74 cfs      |
| Discharge Full   | 25.79 cfs      |
| Slope Full       | 0.000704 ft/ft |
| Flow Type        | Supercritical  |

**Worksheet**  
**Worksheet for Gutter Section**

| Project Description |                    |
|---------------------|--------------------|
| Worksheet           | Gutter Section - 1 |
| Type                | Gutter Section     |
| Solve For           | Spread             |

| Input Data           |                |
|----------------------|----------------|
| Slope                | 0.050000 ft/ft |
| Discharge            | 6.00 cfs       |
| Gutter Width         | 2.00 ft        |
| Gutter Cross Slope   | 0.040000 ft/ft |
| Road Cross Slope     | 0.020000 ft/ft |
| Mannings Coefficient | 0.012          |

| Results           |                     |
|-------------------|---------------------|
| Spread            | 9.03 ft             |
| Flow Area         | 0.9 ft <sup>2</sup> |
| Depth             | 0.22 ft             |
| Gutter Depression | 0.5 in              |
| Velocity          | 7.01 ft/s           |

# Worksheet

## Worksheet for Gutter Section

| Project Description |                    |
|---------------------|--------------------|
| Worksheet           | Gutter Section - 1 |
| Type                | Gutter Section     |
| Solve For           | Spread             |

| Input Data           |                |
|----------------------|----------------|
| Slope                | 0.050000 ft/ft |
| Discharge            | 13.40 cfs      |
| Gutter Width         | 2.00 ft        |
| Gutter Cross Slope   | 0.020000 ft/ft |
| Road Cross Slope     | 0.013000 ft/ft |
| Mannings Coefficient | 0.012          |

| Results           |                     |
|-------------------|---------------------|
| Spread            | 16.50 ft            |
| Flow Area         | 1.8 ft <sup>2</sup> |
| Depth             | 0.23 ft             |
| Gutter Depression | 0.2 in              |
| Velocity          | 7.51 ft/s           |

# Worksheet Worksheet for Gutter Section

| Project Description |                    |
|---------------------|--------------------|
| Worksheet           | Gutter Section - 1 |
| Type                | Gutter Section     |
| Solve For           | Discharge          |

| Input Data           |                |
|----------------------|----------------|
| Slope                | 0.050000 ft/ft |
| Gutter Width         | 2.00 ft        |
| Gutter Cross Slope   | 0.020000 ft/ft |
| Road Cross Slope     | 0.013000 ft/ft |
| Spread               | 12.00 ft       |
| Mannings Coefficient | 0.012          |

| Results           |           |
|-------------------|-----------|
| Discharge         | 5.81 cfs  |
| Flow Area         | 1.0 ft²   |
| Depth             | 0.17 ft   |
| Gutter Depression | 0.2 in    |
| Velocity          | 6.12 ft/s |

# Worksheet Worksheet for Gutter Section

| Project Description |                    |
|---------------------|--------------------|
| Worksheet           | Gutter Section - 1 |
| Type                | Gutter Section     |
| Solve For           | Discharge          |

| Input Data           |                |
|----------------------|----------------|
| Slope                | 0.050000 ft/ft |
| Gutter Width         | 2.00 ft        |
| Gutter Cross Slope   | 0.040000 ft/ft |
| Road Cross Slope     | 0.020000 ft/ft |
| Spread               | 12.00 ft       |
| Mannings Coefficient | 0.012          |

| Results           |           |
|-------------------|-----------|
| Discharge         | 12.27 cfs |
| Flow Area         | 1.5 ft²   |
| Depth             | 0.28 ft   |
| Gutter Depression | 0.5 in    |
| Velocity          | 8.29 ft/s |

**APPENDIX C**  
**STORAGE AND WATER QUALITY TREATMENT VOLUME CALCULATIONS**

8-12-04

# ROUTING

## SCHEME:

|                         |   |                       |
|-------------------------|---|-----------------------|
| OVERLAND FLOW (3)       | } | BASIN @ $\triangle 1$ |
| AC PAVE FLOW (4)        |   |                       |
| BASIN (1)               |   |                       |
| OVERLAND FLOW (5)       |   |                       |
| OVERFLOW $\triangle 14$ |   |                       |

|                   |   |                        |
|-------------------|---|------------------------|
| OVERLAND FLOW (6) | } | BASIN @ $\triangle 14$ |
| AC PAVE FLOW (7)  |   |                        |
| OVERLAND FLOW (8) |   |                        |

## CALCULATIONS

### TREATMENT VOLUME (PAVED ROADWAY)

|                      |         |            |
|----------------------|---------|------------|
| 57,277 SF Lower Road | 4773 CF |            |
| 45,185 SF Upper Road | 3765 CF |            |
|                      | 8538 CF | INFILTRATE |
|                      |         | \$TREAT.   |

### INFILTRATION VOLUME

#### ROOF RUNOFF VOLUME

|         |        |
|---------|--------|
| 3589 SF | 299 CF |
| 5105 SF | 425 CF |
| 3987 SF | 499 CF |

JWD SOILS =  
1" / hr perm.  
(may find more  
permeable in field.)  
per Dave H. / Kleinf.

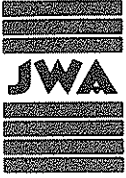
**Infiltration and Storage System Sizing and Capability**  
(use the chart and equation below)

|                              | A     | B     | C     | D | E | F | G | H | I | J | K | L | M | N | TOTAL  |
|------------------------------|-------|-------|-------|---|---|---|---|---|---|---|---|---|---|---|--------|
| surface width (ft)           |       |       |       |   |   |   |   |   |   |   |   |   |   |   | --     |
| surface length (ft)          |       |       |       |   |   |   |   |   |   |   |   |   |   |   | --     |
| surface area (sq. ft.)       | 3589  | 5105  | 5987  |   |   |   |   |   |   |   |   |   |   |   | 14681  |
| volume of runoff (cu. ft.)   | 299.0 | 425.0 | 499.0 |   |   |   |   |   |   |   |   |   |   |   | 1223.4 |
| BMP width (in.)              | 32    | 32    | 32    |   |   |   |   |   |   |   |   |   |   |   | --     |
| BMP depth (in.)              | 24    | 24    | 24    |   |   |   |   |   |   |   |   |   |   |   | --     |
| BMP length (ft.)             | 150   | 220   | 250   |   |   |   |   |   |   |   |   |   |   |   | --     |
| BMP % void                   | 33%   | 33%   | 33%   |   |   |   |   |   |   |   |   |   |   |   | --     |
| soil permeability (in. / hr) | 1     | 1     | 1     |   |   |   |   |   |   |   |   |   |   |   | --     |
| BMP capability (cu. ft.)     | 314.0 | 460.5 | 523.3 |   |   |   |   |   |   |   |   |   |   |   | 1297.9 |
| system installed level (y/n) | Y     | Y     | Y     |   |   |   |   |   |   |   |   |   |   |   | --     |
| * BMP sized correctly?       | YES   | YES   | YES   |   |   |   |   |   |   |   |   |   |   |   | --     |

$$\begin{aligned}
 \text{capacity (cubic feet)} &= \left( \frac{\text{width}''}{12} \times \text{length}' \times \frac{\text{permeability}''/\text{hr}}{12} \right) \\
 &+ \left( \frac{2}{3} \times \frac{\text{depth}''}{12} \times \text{length}' \times \frac{\text{permeability}''/\text{hr}}{12} \right) \\
 &+ \left( \frac{\% \text{void}}{100} \times \frac{\text{width}''}{12} \times \text{length}' \times \frac{\text{depth}''}{12} \right)
 \end{aligned}$$

\* The capability must be greater than or equal to the volume of runoff from the hard surface.

\* All infiltration and storage systems must be installed level.



April 22, 2005

RECEIVED

APR 27 2005

C0310

Correspondence

Suzanne Enslow  
EDAW  
2022 J Street  
Sacramento, CA 95814

Subject: **Cedar Grove Increased Density Alternative Review**

Dear Suzanne:

The following are our comments and analysis of the Reduced and Increased-Density Alternatives in comparison to the previously-prepared analysis of the proposed alternative within our Preliminary Drainage Report of August 2004. These comments are based on alternative figures received on April 4, 2005.

**Location/Direction of Flow Comments**

**Reduced Alternative:**

1. A new access point is to the north, from Donner Road. Road configuration changed from two opposing drainage directions to one.
2. Look at assessor's parcel map for property ownership to the north, to investigate construction of the access road, and associated drainage improvements.
3. No detailed topography to the north. Is the intent to drain north from the property line?
4. Donner Road along the north property line is substantially lower than the property on the western half. Therefore, it is difficult to tell how the new access intersects and ties into Donner Road.
5. There may be a concern if all road drainage flows south. Divert some drainage to the center of the project toward the clubhouse area.
6. How does the road connect to Grey Lane? Does it go upslope from Clubhouse?
7. It may be difficult to divert runoff to a basin in the lower southeast corner due to positions of buildings and new low point of road.

### **Increased-Density Alternative**

1. Note the new access point, Grey Lane.
2. The road may slope toward the south on each side.
3. Maintain the opportunity to drain the roads to the southeast; however, the area for location of the basin at point of interest (POI) 14 appears to have been eliminated.
4. Possibly another basin could be provided at a point commensurate with POI 14 near the POI 3 and 4, and either toward the rear of Building 13 or between Building 13 and Building 11. Old numbers may not depict building numbers on what was sent.
5. Use of the area described in (4) above for a basin, is limited due to slopes and may be in a fill area.
6. Define a buffer area from property line and indicate whether the space may include detention facilities.
7. Overflow is limited to a point or ditch system that would be necessary to the south to discharge to U.S. Highway 28.
8. Runoff from the northeastern quarter of the property could be directed to Grey Lane, but required area for a detention facility area is not available, and may require underground detention.
9. Possibly the interior of roadway system could be utilized for detention with overflow via piping to outlet or discharge point. For example, overflow the northern one half of parcel via pipe to Grey Lane, and overflow the southern one half of parcel via a pipe to a detention basin overflow, then discharge as in (7) above.

### **Quantity of Flow Comments**

#### **Reduced-Density Alternative**

1. There is room to put several discharge points for treatment. One half of parcel may continue to discharge to a basin which overflows to Grey Lane. The other half of the parcel may continue to discharge to Toyon and to a southeast basin.

2. Coverage reduced by \_\_\_\_\_% (cannot read text on this page of PDF)

- Discharge Q is less than proposed alternative
- Storage Volume is less than proposed requirement

### **Increased-Density Alternative**

1. Coverage increased by 2.3% over proposed alternative

- Insignificant increase in Discharge Q
- Insignificant increase in storage volume

Study performed for proposed alternative is over-conservative and assumes up to 50% on-site impermeable land coverage.

2. Increased overall infiltration requirements by 2.3%

- If building footprints are larger, they will result in more or larger infiltration trenches. An alternative design would be for more discharge points along roadway, or the use of underground infiltration chamber systems if the volume of runoff to infiltrate at each specific location exceeds soil capability.
- Volume increase creates a need for larger detention facilities and an increase in area or depth.

### **Summary**

In summary, the Proposed Density Alternative Analysis in the Preliminary Drainage Report is still valid in that the areas estimated for use for infiltration and detention should be sufficient even for the proposed Increased Density Alternative. The Proposed Alternative was analyzed with a highly-conservative approach. However, attention to the required detention basin areas should be reviewed on the new the plan layout for both the reduced and increased density alternatives, as the designated areas have been minimized. Excavation depths, sidewall slopes, and distance from structures should be addressed in design to find the actual available area to locate a detention basin. A final drainage analysis will determine if the volumes of storage basins are sufficient.

Treatment BMPs for the roadway should be designed similarly. An increase in the impervious surface of the parking and roadway creates the need for more areas which the ability to infiltrate the 20-year, 1-hour storm volume or to treat and route the runoff from the paved surfaces to infiltration basins or galleries. More paved surfaces results in more infiltration volume required by TRPA. A final geotechnical analysis should be done to determine soil permeability in areas

designated for infiltration.

Additionally, all alternatives concentrate the surface runoff, which must be diverted to new discharge points acceptable to Placer County. Once developed, the site does not allow for the overland flow and shallow concentrated flow allowed currently along the southern border of the property. Detention basins to control the discharge to a pre-developed flow from the site further concentrates the flow at the basin outlet and overflow devices.

At this time, further detailed analysis is not warranted. If there are any additional questions, please call us at our Zephyr Cove office (775) 588-7178.

Sincerely,



Cindy Neisess  
Engineer Intern



Jennifer G. Roman, P.E.  
Senior Engineer

CN/JGR/jal



JWA CONSULTING ENGINEERS, INC.

RECEIVED

APR 29 2005

April 27, 2005

C0310  
Correspondence

Suzanne Enslow  
EDAW  
2022 J Street  
Sacramento, CA 95814

Subject: **Cedar Grove Additional Alternative Review**

Dear Suzanne:

The following are our comments and analysis of the For Sale Moderate Income Alternative at 30% coverage in comparison to the previously-prepared analysis of the Proposed Density Alternative within our Preliminary Drainage Report of August 2004. These comments are based on the alternative figure received on Tuesday, April 26, 2005.

**Location/Direction of Flow Comments**

1. A new access point is to the north, from Donner Road.
2. Look at assessor's parcel map for property ownership to the north, to investigate construction of the access road, and associated drainage improvements.
3. No detailed topography to the north. Is the intent to drain north from the property line?
4. Donner Road along the north property line is substantially lower than the property on the western half. Therefore, it is difficult to tell how the new access intersects and ties into Donner Road. In this alternative it may be possible to meet grade at the midpoint of the eastern one-half property line.
5. The drainage pattern is similar to the proposed alternative, and the development is accessed also by Grey Lane and Toyon Road. Divert some drainage to the center of the project toward the open area for infiltration and for possible detention in a basin.
6. Possible locations of detention basins have shifted toward the west and the southwest. It is difficult to tell without preliminary proposed grading which areas are intended for cut or fill.
7. Foundation depths should be checked versus allowable cuts, per the Tahoe Regional Planning Agency (TRPA) depths of excavation approved for this project.

8. It may be difficult to divert runoff to a basin in the lower southeast corner due to positions of buildings. Final grading plans should drain the streets toward detention facilities.
9. Maintain the opportunity to drain the roads to the southeast; however, the area for location of the basin at Point of Interest (POI) 14 appears to have been eliminated.
10. Overflow from a southwesterly located basin is limited to a point or ditch system that would be necessary to the south to discharge to U.S. Highway 28.
11. Runoff from the northeastern quarter of the property could be directed to Grey Lane, but required area for a detention facility is not available, and may require underground detention.

#### **Quantity of Flow Comments**

1. There is not a lot of opportunity for small infiltration basins along the roadway in the 50% coverage area to overflow to accommodate larger flows from larger storm events. Piping may be necessary to convey drainage under sidewalks.
2. Storage volume for associated impervious area in the 50% coverage area is increased. This may be conveyed to a larger infiltration basin in the area between the affordable and moderate housing. However, this area should be utilized for detention and designed for an open area rather than limited by surface infiltration basins. Underground infiltration could be designed with overflow runoff directed to the street and discharged to Grey Lane. Another option could utilize conveyance swales to route overflow runoff to a large basin in the southwest corner of the property.
3. If building footprints are larger, they will result in more or larger infiltration facilities.
4. An alternative design would be for more discharge points along roadway, or the use of underground infiltration chamber systems if the volume of runoff to infiltrate at each specific location exceeds soil capability.
5. The additional coverage increases the runoff volume and creates a need for larger detention facilities.

#### **Summary**

In summary, the Proposed Density Alternative Analysis in the Preliminary Drainage Report is still valid in that the areas estimated for use for infiltration and detention should be sufficient even for the proposed Increased Density Alternative. The Proposed Alternative was analyzed with a highly-conservative approach. Attention to the required detention basin areas should be

reviewed on the new plan layout for any increase in density in each localized area. Excavation depths, sidewall slopes, and distance from structures should be addressed in design to find the actual available area to locate a detention basin. A final drainage analysis will determine if the volumes of storage basins are sufficient.

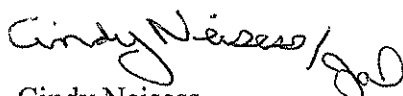
Treatment Best Management Practices (BMPs) for the roadway should be designed similarly. An increase in the impervious surface of the parking and roadway creates the need for more areas with the ability to infiltrate the 20-year, 1-hour storm volume or to treat and route the runoff from the paved surfaces to infiltration basins or galleries. More paved surfaces result in more infiltration volume required by TRPA. A final geotechnical analysis should be performed to determine soil permeability in areas designated for infiltration.

Additionally, all alternatives concentrate the surface runoff, which must be diverted to new discharge points acceptable to Placer County. Once developed, the site will not allow for the overland flow and shallow concentrated flow allowed currently along the southern border of the property. Detention basins to control the discharge to a pre-developed flow from the site further concentrates the flow at the basin outlet and overflow devices.

Any further increased impervious coverage such as an overall 50% coverage alternative, will follow the above comments and be subject to similar requirements of the analysis previously performed for the Increased Density Alternative, in a letter addressed to you on April 22, 2005.

At this time, further detailed analysis of this alternative is not warranted. If there are any additional questions, please call us at our Zephyr Cove office.

Sincerely,



Cindy Neisess  
Engineer Intern



Jennifer G. Roman, P.E.  
Senior Engineer

CN/JGR/jal



C0310

Correspondence

May 6, 2005

Suzanne Enslow  
EDAW  
2022 J Street  
Sacramento, CA 95814

Subject: **Cedar Grove Apts EIR/EIS - Hydrology**

Dear Suzanne:

The following are our comments and questions regarding the input from Placer County within your recent e-mail of May 3, 2005:

1. A 10-year storm event must be used for design of drainages and dedicated drainage facilities and systems. This is also a Tahoe Regional Planning Agency (TRPA) requirement. We are unfamiliar with any requirement by Placer County as far as infiltration of the 10-year storm event. Infiltration may be an alternative means of reducing the amount of discharge from a site, but must be approved by the Flood Control District.
2. Infiltration of the 20 year-hour storm event is required by Lahontan, and TRPA concurs.
3. Although we agree with Placer County that the SWMM requires detention or attenuation of the 100- year peak flow, it should be clarified that discharge is allowed for pre-development levels to existing points of discharge.

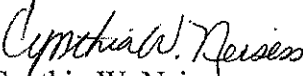
Generally, it is assumed that the pre-development flows do not impact the downstream facilities. However, Placer County should be responsible for analyzing whether the downstream facilities are sufficient in their current condition. The concern in our preliminary drainage report is for re-routing the concentrated discharge or outflow of the detention facilities to possibly a new discharge point downstream to the south that currently receives overland flow only, thereby changing the manner of flow. An off-site drainage system should be considered by the County in this scenario for routing of overflow and discharges from the detention facilities on a regional basis.


The downstream limit of the hydrologic study provided within the JWA Consulting Engineers, Inc. (JWA), scope of services is the point beyond which changes in peak flows would not be measurable. (See the PCSWMM Section VII.C.3.) If the detention facilities are constructed for the volumes described in the Preliminary Drainage Report, there would be no increase in peak flows at the discharge locations and, therefore, no off-site studies are required. However, varying from those preliminary design parameters would require that the project proponent seek approval and coordinate with the County on additional analysis of alternative drainage systems or plans. For instance, if the preferred alternative seeks approval of a lesser size detention facility or of increased discharge to downstream off-site facilities, input from the County will be required.

Finally, in accordance with the PCSWMM Section VI.B.2.a., since the watershed is less than 200 acres, the developer is required to only submit information in regards to local drainage. Roadways and conveyance to detention facilities must meet the County requirements for 10-, 25-, and 100-year storm events. These commitments should be a condition of approval and must become design constraints within the final design of the project.

Certainly, additional information may be requested to supplement the preliminary drainage report, within JWA's requirement to respond to comments from your Administrative Draft. Please contact me at our Zephyr Cove office if you have questions or comments, or if you require additional information.

Sincerely,

  
Cynthia W. Neisess  
Engineer Intern

  
Jennifer G. Roman, P.E.  
Senior Engineer

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